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CONTENTS

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RP230

STRENGTH OF WELDED SHELF-ANGLE CONNECTIONS

By James H. Edwards, H. L. Whittemore, and A. H. Stang

ABSTRACT

Shelf angles for transferring loads from floor beams to columns in steel-frame buildings were arc welded to H-section steel columns. The angles on the different specimens varied in width, thickness, and length. In some of the specimens the vertical leg of the shelf angle was placed against the web or flange of the column and fastened with fillet welds along one or more of the edges of contact. These types of specimens were: (A) Single weld at heel of angle; (B) welds at heel and toe of angle; (C) welds at ends of angle; (D) welds at heel and ends of angle; and (E) welds at heel, toe, and ends of angle. However, in the specimens of type F the angle was made of such a length that it filled snugly between the flanges of the column and was fastened to the column with fillet welds at both ends of the angle. The inner face of the vertical leg of the angle was placed 2 inches from the center line of the column web.

The specimens were tested to destruction in the 10,000,000-pound testing machine of the National Bureau of Standards. They were placed in the machine in an inverted position so that the outstanding legs of the angles rested on 6 by 6 inch steel bearing blocks placed on the lower platen of the machine. In testing all of the specimens, except type F, the inside faces of the bearing blocks were placed one-half inch from the face of the column to simulate end clearance in the floor beam. For type F the bearing blocks were placed under the entire width of the horizontal legs of the angles.

Increasing the width of the vertical leg of the angle increased the maximum stress for specimens welded only at the ends, type C. There were no definite indications that the width of the vertical leg of the angles affected the strength of the other types of specimens. Neither the thickness nor the length of the angles appeared to have much effect on the unit strength of the specimens.

Shelf-angle connections in designing which the stresses are those recommended by the Code for Fusion Welding and Gas Cutting in Building Construction, prepared by the American Welding Society, will have a factor of safety of about four, provided the welds are reinforced as much as the welds on these specimens.

CONTENTS

I. INTRODUCTION

1. PRELIMINARY

The increased use of welding in fabricating steel structures has given rise to many questions regarding the strength of welded connections. In the past most of the tests have been made on relatively small specimens in which the distribution of stress is simple and for which there is an exact method of stress analysis. These have served to establish safe values of stress to be used in design as well as methods of supervision and inspection to insure that good welds are produced. There still remains the need of investigating full-sized connections subjected to the more complex conditions of stress which are present in an actual structure. Definite knowledge of their behavior under load makes it possible to design welded connections which are equally strong in resisting the various conditions of stress to which they may be subjected.

2. PURPOSE

These tests were made to determine the strength of welded connections of the seat-angle type for use in steel-frame buildings. In this type of connection, angles are welded to the outside of the flanges or to the inside of the flanges of the rolled steel columns as shown at *A* and *C* in Figure 1. The ends of the beams and girders which carry the floor are supported on these angles. The effect of changing the dimensions of the shelf angles and the location of the welds with respect to the angles were studied.

3. ACKNOWLEDGMENTS

The tests were made at the National Bureau of Standards. The American Bridge Co. furnished the specimens which were designed by James H. Edwards, chief engineer. O. E. Hovey, assistant chief engineer, and other members of the company's engineering staff assisted in making the tests. Prof. Elmer O. Bergman, research associate, made the analysis of the test data and edited the manuscript.

II. SPECIMENS

1. MAKE-UP

A test specimen consisted of a pair of angles welded to opposite faces of an H-section steel column as shown either at *A* or at *C* in Figure 1. The columns were Carnegie beam sections 3 and 8 feet long and 10 (CB104) (CB105), 12 (CB126), or 14 (CB146) inches deep. To reduce the number of columns required, two pairs of angles were welded to each column as shown in Figure 1. As the columns were not, except in a few cases, visibly deformed after the tests, the size of the columns had little effect upon the strength of the shelf angles. As each of the specimens failed through the welds, the properties of the material in the angles did not appear to affect the results of these tests.

According to the manner of attaching the shelf angle to the column, there were two classes of specimens. For class 1, the vertical leg of the angle was placed in contact with the outside of the flange of the column (see *A–A*, fig. 1) and attached with various combinations of welds as shown for types A, B, C, D, and E (fig. 2). For class 2,

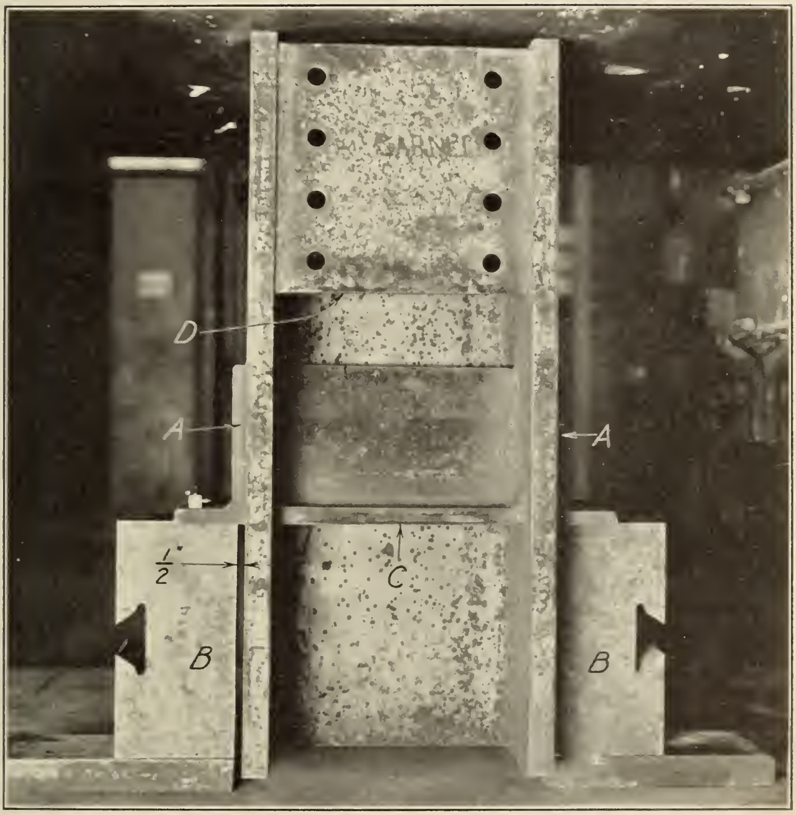

FIGURE 1.—*Column having welded shelf-angle connections*

(Patent No. 1757359, May 6, 1930, James H. Edwards, "Steel Building, Frame Construction.") Specimen in machine ready for testing. *A–A*, Shelf angles welded to the outside of the flange of the H-section. Specimens *A* to *E*, inclusive, were of this type. *B–B*, steel blocks through which the load was applied. As floor beams do not extend over the entire horizontal leg of the angle, spaces were left between the blocks and the column. *C*, shelf angles welded at the ends of the angles to the inside of the flanges of the H-section. Only one is shown as the other angle is on the other side of the column. The distance between the flange and the web ranged from 1⅛ to 1⅝ inches. The steel blocks, *B–B*, were used for loading these angles. They were placed under the entire width of the horizontal legs of the angles. The plate shown at *D* was used in another investigation.

the angle was of such a length that it fitted snugly between the inside faces of the column flanges as shown at *C* (fig. 1), and type F (fig. 2), to which it was fastened by a horizontal and a vertical weld at each

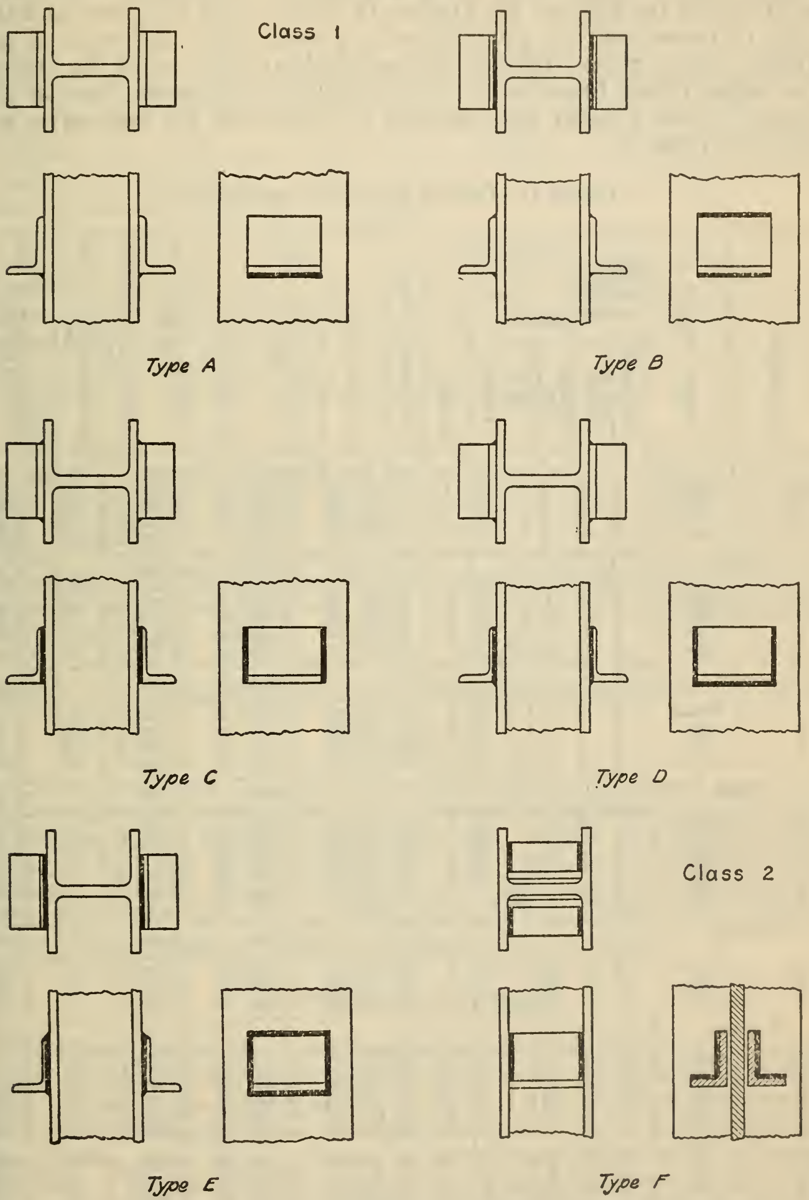

FIGURE 2.—*Drawing of each of the six types of specimens*

end. The inner face of the angle, *C* (fig. 1), was 2 inches from the center line of the web. The distance between the flange of the angle and the web of the column ranged from 1⅛ to 1 ⅝ inches, depending upon the size of the column.

The six types of welds are shown in Figure 2 and may be described as follows: A, horizontal welds along the heels of the angles; B, horizontal welds along the heels and toes of the angles; C, vertical welds along the ends of the angles; D, welds along the heels and the ends of the angles; E, welds along the heels, toes, and ends of the angles; and F, horizontal and vertical welds at each end of the angles, the angles fitting between and being welded to the inside faces of the flanges. The number and position of the welds for each type are given in Table 1.

TABLE 1.—*Tests of shelf-angle connections*

Type and No.	Manner of support	Number of welds on each angle: Horizontal	Number of welds on each angle: Vertical	Dimensions of angles: Width—Vertical leg	Dimensions of angles: Width—Horizontal leg	Dimensions of angles: Thickness	Dimensions of angles: Length	Total length of weld	Maximum load	Maximum load per unit length	Average throat	Maximum stiess on throat area	Factor of safety on throat area[1]	Average width of fracture	Maximum stress on area of fracture
				In.	In.	In.	In.	In.	Lbs.	Lbs./in.	In.	Lbs./in.[2]		In.	Lbs./in.[2]
A-1	Partial	1	0	4	4	5/8	8	16	363,000	22,700	0.31	[2] 73,000	5.6	7/16	51,900
2	do	1	0	4	4	5/8	8	16	412,500	25,800	.29	[2] 89,000	6.8	11/32	75,000
3	do	1	0	6	4	3/4	8	16	421,500	26,300	.27	[2] 98,000	7.5	3/8	70,000
4	do	1	0	6	4	3/4	8	16	353,000	22,100	.27	[2] 82,000	6.3	11/32	64,300
Average		----	----	----	----	----	----	----	--------	24,200	----	[2] 85,500	----	-----	65,300
B-1	Partial	2	0	4	4	5/8	8	32	556,000	17,400	0.26	67,000	4.8	7/16	39,800
2	do	2	0	6	4	3/4	8	32	473,000	14,800	.22	67,000	4.8	3/8	39,200
3	do	2	0	7	3 1/2	7/8	10	40	613,000	15,300	.28	55,000	3.9	3/8	40,800
4	do	2	0	8	3 1/2	7/8	10	40	535,000	13,400	.31	43,000	3.1	7/16	30,600
Average		----	----	----	----	----	----	----	--------	15,200	----	58,000	----	-----	37,600
C-1	Partial	0	2	4	4	1/2	8	16	173,000	10,800	0.27	40,000	3.5	11/32	31,400
2	do	0	2	6	4	5/8	8	24	313,000	13,000	.24	54,000	4.8	11/32	38,100
3	do	0	2	7	3 1/2	3/4	10	28	396,000	14,100	.23	61,000	5.4	3/8	37,600
4	do	0	2	8	3 1/2	3/4	10	32	446,000	13,900	.23	61,000	5.4	5/16	44,800
Average		----	----	----	----	----	----	----	--------	13,000	----	54,000	----	-----	38,000
D-1	Partial	1	2	4	4	3/4	6	28	495,000	17,700	0.31	57,000	4.7	3/8	47,100
2	do	1	2	4	4	3/4	6	28	491,000	17,500	.29	61,000	5.0	11/32	51,000
3	do	1	2	4	4	3/4	8	32	514,000	16,100	.28	57,000	4.7	3/8	42,900
4	do	1	2	6	4	7/8	8	40	521,000	13,000	.26	50,000	4.2	3/8	34,700
5	do	1	2	7	3 1/2	3/4	10	48	596,000	12,400	.28	44,000	3.7	3/8	33,100
6	do	1	2	8	3 1/2	1	10	52	689,500	13,300	.27	49,000	4.1	13/32	32,600
Average		----	----	----	----	----	----	----	--------	15,000	----	53,000	----	-----	40,200
E-1	Partial	2	2	4	4	3/4	8	48	676,000	14,100	0.27	52,000	4.0	3/8	37,600
2	do	2	2	6	4	7/8	8	56	799,000	14,300	.27	53,000	4.1	3/8	38,200
3	do	2	2	8	3 1/2	1	10	72	1,062,000	14,800	.26	57,000	4.4	3/8	39,500
Average		----	----	----	----	----	----	----	--------	14,400	----	54,000	----	-----	38,400
F-1	Full	2	2	4	4	5/8	7 3/4	27	316,500	11,700	0.24	49,000	4.0	11/32	34,100
2	do	2	2	4	4	3/4	7 3/4	26	359,000	13,800	.24	58,000	4.7	11/32	40,200
3	do	2	2	6	4	5/8	7 3/4	35	504,000	14,400	.27	53,000	4.4	13/32	35,400
4	do	2	2	6	4	3/4	8 1/4	34	488,000	14,300	.34	42,000	3.5	13/32	35,300
5	do	2	2	6	4	3/4	8 1/4	34	480,500	14,100	.32	44,000	3.7	13/32	34,800
6	do	2	2	6	4	5/8	9 1/4	35	447,500	12,800	.25	51,000	4.3	11/32	37,300
7	do	2	2	6	4	3/4	9 1/4	34	433,000	12,700	.24	53,000	4.4	3/8	33,900
8	do	2	2	8	3 1/2	3/4	12 1/4	40	615,000	15,400	.27	57,000	4.8	5/16	49,300
9	do	2	2	8	3 1/2	7/8	12 1/4	39	526,000	13,500	.27	50,000	4.2	11/32	39,300
Average		----	----	----	----	----	----	----	--------	13,600	----	51,000	----	-----	37,700

[1] Shelf angles designed by the method described in IV, Test Data.
[2] These values for maximum stress are much higher than the value found by the analysis given in V-1, Discussion, Welds, which are, in all probability, more nearly the actual stresses in the welds.

2. WELDS

The welder passed the qualification tests of the Structural Steel Welding Committee, American Bureau of Welding.[1] The welding rod was three-sixteenths inch in diameter (carbon 0.17 and manganese 0.52 per cent). The arc characteristics were d.c., average amperes 190, average volts 19. The average ultimate strength of all the 20 qualification specimens was 55,520 lbs./in.^{2} and the minimum 43,990 lbs./in.^{2} The average strength of the 12 specimens welded in the horizontal position was 56,990 lbs./in.^{2} and in the vertical position 53,330 lbs./in.^{2} The required average was 45,000 lbs./in.^{2} and the minimum was 40,000 lbs./in.^{2}

All the welds on these specimens were of the fillet type; that is, they were triangular welds, joining two surfaces approximately perpendicular to each other. The nominal size of the welds was 5/16 inch, the size of a fillet weld being the designed length of its legs.[2] The actual sizes varied from 1/4 inch and 7/16 inch for one weld on specimen F–5 to 1/2 inch and 5/8 inch for one weld on specimen F–7. They were reinforced so that the actual throat was about equal to the leg of the weld.

The throat of a fillet weld, as defined in the Code for Fusion Welding and Gas Cutting in Building Construction,[3] is the normal distance from the root of the weld to the hypotenuse of the largest isosceles right triangle that can be constructed in the cross section of the fillet weld. The throat of these specimens was computed by multiplying the width of the narrower side of the weld by 0.707. Measurements on many of the welds showed that the actual throat was equal to the shorter leg of the weld.

The lengths of the welds are the corresponding dimensions of the angles. No deductions were made for craters at the ends of the welds.

3. ANGLES

From stock sizes of rolled steel angles, four sizes, having about the same width of horizontal leg, were selected to obtain a considerable difference in the width of the vertical leg. Angles with 4 and 6 inch vertical legs had 4-inch horizontal legs; those with 7 and 8 inch vertical legs had 3½-inch horizontal legs. The angles varied in length from 6 to 12½ inches and in thickness from ½ to 1 inch. The dimensions of the angles are given in Table 1.

III. TEST PROCEDURE

The specimens were tested to destruction in the 10,000,000 pounds capacity hydraulic testing machine at the National Bureau of Standards. They were placed in the machine in an inverted position so that the outstanding legs of the angles rested on 6 by 6 inch steel bearing blocks, *B–B* (fig. 1) placed on the lower platen of the testing machine. This platen is supported by a spherical bearing. After the specimen was placed in the machine, the lower platen was adjusted to

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[1] Specifications for Test Specimens, section B, Qualification of Welders, Part I, Direct Current Metal Arc Process, American Bureau of Welding, 33 West Thirty-ninth Street, New York, N. Y.

[2] Paragraph 75, Weld Size, p. 16, Welding and Cutting Nomenclature, Definitions and Symbols, American Welding Society, 33 West Thirty-ninth Street, New York, N. Y., November, 1929.

[3] Code for Fusion Welding and Gas Cutting in Building Construction, Part A, Structural Steel, Edition 1928, American Welding Society, 33 West Thirty-ninth Street, New York, N. Y.

bring the upper surface of the column parallel with the upper platen; thus, the applied load was distributed approximately uniformly over the column. In all but two specimens (D–1 and D–2) the angles extended beyond the ends of the blocks. Figure 1 shows a specimen in the machine ready for testing.

The methods used in loading the specimens were designed to approximate the loading of a similar connection in an actual structure. In a structure the load on the shelf angle is the end reaction of the floor beam. To facilitate erection, clearance is provided between the ends of the beam and the faces of the columns. Figure 1 shows the method used in loading the specimens, having the vertical leg of the angle against the face of the column (types A, B, C, D, and E). The distance between the face of the bearing block and the face of the column was one-half inch so as to load the angle approximately as it would be loaded by a floor beam having clearance at the ends. This method of loading is referred to in this paper as "partial support."

For specimens of type F, having the angles welded at their ends to the inside faces of the column flanges, the supporting blocks were placed under the entire width of the horizontal legs of the angles, because in an actual structure the clearance at the ends of the beam would be less than the distance between the angle and the web of the column. This method of loading is designated "full support."

IV. TEST DATA

The values of the maximum loads and the corresponding values of the maximum loads per unit length of weld and maximum stresses are given in Table 1. In this table the data for each type of specimen are arranged according to the width of the vertical leg of the angles beginning with the narrowest.

The computed maximum load per unit length of weld is the maximum load divided by the sum of the lengths of all the welds. The computed maximum stress on the throat area is the maximum load divided by the sum of the throat areas of all the welds; that is, the average throat multiplied by the sum of the lengths of all the welds.

For computing the factor of safety, it was assumed that the forces on the shelf angles acted in the faces of the columns. For types A, B, C, D, and E it was assumed that the vertical welds were under shearing stress and the horizontal welds at the heel of the angles under tension and at the toe under compressive stress. Similarly, for type F all the vertical welds were assumed to be under shearing stress and the horizontal welds under tensile stress because the angle deflected.

The stresses recommended by the Code for Fusion Welding were used: that is, shear 11,300, tension 13,000 and compression 15,000 lbs./in.^{2} on the section through the throat of the weld. Dividing the observed maximum load for each specimen by the safe working load computed in this way, a factor of safety was obtained; these values are given in Table 1. A factor of safety of four is usually considered safe for statically loaded steel structures.

This method of design is admittedly the roughest kind of an approximation. Under V–1. Discussion, Welds, a somewhat closer analysis of some of the types of specimens is attempted. These seem to indicate that the computed values of the maximum stress given in Table 1 are higher than the actual stresses, especially for type A specimens.

As the actual throats were in most cases considerably larger than the throat computed in accordance with the Fusion Welding Code, and some of the welds did not rupture through the throat, the actual width of the fracture was measured. These values are given in Table 1. The maximum stress on fractured area was obtained by dividing the maximum load by the sum of the fractured areas of all the welds; that is, the average width of the fracture multiplied by the sum of the lengths of all the welds.

A detailed account of the behavior of the specimens follows.

Type A.—Horizontal welds along the heels of the angles. The angles of specimens 1 and 3 scaled. The specimens failed suddenly by rupture across the throat of the weld at the maximum load.

Type B.—Horizontal welds along the heels and toes of the angles. The heel welds of specimen 4 scaled and those of specimens 1 and 3 cracked. The maximum loads were only slightly greater than those which caused scaling or cracking. The welds at the heel failed across the throat and those at the toe failed along the leg on the column. The horizontal legs of the angles bent, the thickest angles bending the least. The horizontal legs of the angles of specimen 1 started to shear at the corners of the bearing blocks.

Type C.—Vertical welds along the ends of the angles. Failure occurred by rupture of the weld at the throat, fracture beginning at the lower or heel end of the weld. The outstanding legs of the angles bent, the thickest angles bending the least.

Type D.—Welds along the heels and the ends of the angles. The angles of specimens 2, 3, 5, and 6 scaled. The welds along the heel cracked at the maximum load, after which the load decreased very rapidly. Failure occured by rupture of the welds at the throat.

Type E.—Welds along the heels, toes, and ends of the angles. The angles of specimens 1 and 2 and the column flanges of specimen 3 scaled. In general, the welds fractured across the throat at the heel and the ends of the angles and along the leg on the column at the toe of the angles. The outstanding legs of the angles of specimens 1 and 2 bent.

Type F.—Both legs of angles welded to the inside faces of the flanges. The welds scaled and the angles bent. The vertical legs of the angles of specimens 4, 6, 8, and 9 also scaled. Failure of the welds began at the toe of the angles. The column flanges of specimens 1, 2, and 7 bulged outward. One angle of specimen 6 sheared at the heel between the edges of the bearing block and the flanges of the columns. Failure occurred by rupture of the vertical welds at the throat and of the horizontal welds along the leg on the column.

V. DISCUSSION

1. WELDS

The forces acting on the shelf angle for a specimen of type A are shown in Figure 3. The shelf angle on each side of the column is shown to emphasize the fact that the forces are symmetrically arranged with respect to the column. The column and the floor beams are shown by dashed lines. It should be remembered that in this figure the members are shown inverted with respect to the position they would occupy in a structure because the specimens

were tested in this position: P is the force applied by the floor beams, S and T are components of the stress acting across the cut section between the weld and the column, C is the resultant compressive force which the column exerts on the vertical leg of the angle, and F_1 and F_2 are frictional forces. The weld will fail when the stress, of which S and T are the components, becomes greater than the strength of the weld. Both F_1 and F_2 reduce the values which S and T would have if no friction were present.

The authors are indebted to O. E. Hovey for the following analysis of the stresses in the specimens of type A.

As under the action of the force P exerted by the bearing block, the outstanding leg of the angle would deflect, the action line of P is probably outside but close to the outer surface of the vertical leg of the angle about in the position shown in Figure 3.

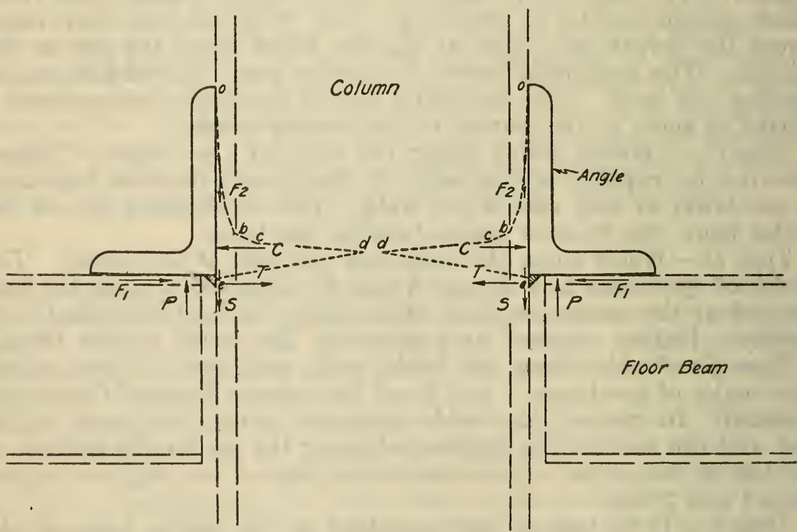

FIGURE 3.—*Free-body diagram showing the forces acting on a shelf angle and the weld of a specimen of type A having a weld at the heel of the angle*

The pressure of the vertical leg of the angle against the flange of the column would cause bending stresses in this leg of the angle which increase from zero at the toe to a maximum at the fillet. Because of the deflection of the vertical leg, the pressures would be expected to vary approximately inversely as the cube of the distance from the fillet. Opposite the fillet the stress distribution is far from simple and need not be discussed. Below the fillet the pressure distribution would be expected to follow closely the compressive stress in a simple beam taking the centroid of the weld as the neutral axis.

An approximate pressure distribution curve is shown at *o b c d e* (fig. 3). The action line of the resultant C is probably slightly above the upper surface of the outstanding leg of the angle as shown.

Knowing, approximately, the action lines of the forces P and C and assuming a coefficient of friction (say 0.3), the values of the forces and of the stress on the weld may be computed. The stress computed in this way is about 60,000 lbs./in^2.

The values of the maximum stress computed on the throat area for specimens of type A given in Table 1 appear to be much higher than the actual stresses and somwehat higher than those computed on the fractured area. A shelf angle designed by the method described in IV, Test Data, would, therefore, be safe. This is indicated, also by the high factors of safety given in Table 1.

The forces acting on a specimen of type B are shown in Figure 4. The forces are similar to those acting on specimens of type A (therefore the notation is the same) with the addition of the shearing force S_T on the toe of the angle. Due to the force S_T, the magnitude of the force S_H is less than in specimens of type A.

The low values of the factor of safety for specimens B–3 and B–4, particularly the latter, raise the question as to the safety of this type of shelf angle when designed in this way. It should be pointed out that if specimen B–4 had been the only one tested, it would have been concluded that the factor of safety of this type of shelf angle was only 3.08.

A discussion of the forces acting on specimens of type C is given later under V–2 (*a*), Angles, Width. For these specimens the stress in the welds is, for the most part, shear. Shelf angles of this type designed as described in IV, Test Data, may be satisfactory, although specimen C–1 showed a low factor of safety.

For types D, E, and F the variation in the number and position of the welds and in the width and length of the angles is too complex to warrant an analysis based on the limited number of tests.

Shelf angles of these types designed as described in IV, Test Data, might be safe, although specimens D–5, E–1, and F–1, 4, and 5 showed factors of safety less than four.

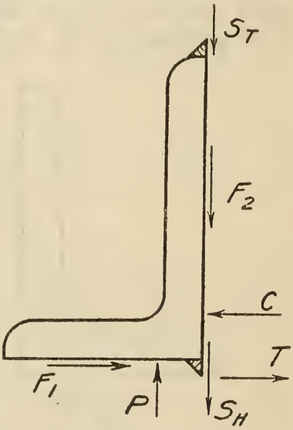

FIGURE 4.—*Free-body diagram showing the forces acting on a shelf angle and the welds of a specimen of type B having welds at the heel and the toe of the angle*

The bending of the shelf angles of type F probably caused high local stresses in the welds resulting in the low factors of safety.

2. ANGLES

(a) WIDTH

There are no definite indications that the width of the vertical leg of the angles affected the strength of the specimens of types A, D, E, and F. For type B there is a decrease and for type C an increase in the maximum stress with an increase in the width of the angle.

For specimens of type B the distribution of the load between the heel and toe welds depends on the stiffness of the vertical leg of the angles. The greater the width of the vertical leg, the larger will be the deformation. As a result the heel weld carries a larger proportion of the load for the lower loads. At and near the maximum load, however, the load must be distributed approximately equally because

the welds in all probability have sufficient ductility to detrude at the maximum load, enough to equalize the loads. Tests upon longitudinal fillet welds, similar to these, showed that the detrusion was about 0.1 inch before the load decreased appreciably from the maximum. For the angles having an 8-inch vertical leg a detrusion in the heel weld of only 0.01 inch more than the toe weld would appear to be all that is necessary to equalize the loads. This detrusion must occur in a direction prependicular to the direction of the weld, and might not reach the value found for the detrusion in the direction of the weld.

The forces acting on a shelf angle of specimens of type C are shown in Figure 5. P is the external force applied to the angle, S is the sum of the friction and the shearing force on the welds, C is the resultant compressive force between the column and the toe of the angle and upper portion of the welds, T is the resultant tensile force on the lower portion of the welds. The angle is in equilibrium under the

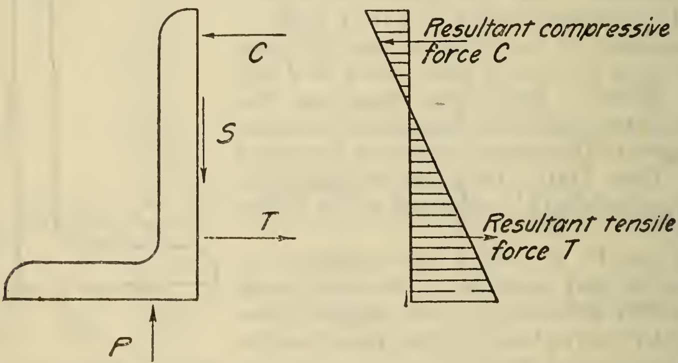

FIGURE 5.—*Free-body diagram showing the forces acting on a shelf angle and the welds of a specimen of type C having welds at the ends of the angle*

action of these forces which form two opposing couples. If the width of the vertical leg of the angle is, for example, doubled and the load P is also doubled (the average shearing stress in the weld remaining the same), the magnitude of the forces T and C will not change since their moment arm is doubled. As a result the tensile stress in the lower portion of the weld will be reduced to one-half its former value, while the shearing stress will remain unchanged. The resultant stress in the lower portion of the weld will, therefore, be decreased as the width of the vertical leg of the angle increases, and a greater maximum load per unit length will be required to cause failure. This is approximately confirmed by the increase in the value of the maximum stress (type C, Table 1) with an increase in the width of the vertical leg of the angle.

(b) THICKNESS

No definite conclusions can be drawn regarding the effect of the thickness of the angles on the strength of the connection. In type C there is an increase in strength with increase of width. The higher loads carried by the wider angles may be due, in part, to their greater thickness.

If we exclude the specimens of type A for which the thickness of the angles, obviously, had little effect upon the maximum stress, we find the following average maximum stresses on the throat area: Angles ½ inch thick, 40,000 lbs/in.²; ⅝ inch thick, 54,800; ¾ inch thick, 55,000; ⅞ inch thick, 50, 200; and 1 inch thick, 53,000. It is evident that the thickness of the angles has practically no effect upon the maximum stress, at least for angles thicker than ½ inch.

(c) LENGTH

Since the length of the angles ranged from 6 to 12 inches and the load was applied only over the middle 6 inches, there were bending stresses in the direction of the length of the angle. These bending stresses might be expected to lower the values of the maximum stresses given in Table 1 for the longer angles.

The angles for specimens of type A were all 8 inches long so no conclusions can be drawn from the results on these specimens. Considering only specimens of types B, C, D, and E, the average maximum stress on the throat area for the angles 6 inches long is 59,000 lbs/in.²; for the angles 8 inches long, 55,000; and for the angles 10 inches long, 52,900. These values tend to confirm the assumed effect of bending stresses, but the differences are small and may have been caused by other variables.

For specimens of type F, the average maximum stress on the throat area for the 7¾-inch angles is 53,300; for the 8¼-inch angles, 43,000; for the 9¼-inch angles, 52,500; and for the 12¼-inch angles, 53,500 lbs./in.² It is evident that for these specimens the length of the angles has very little effect upon the stress. As they were supported only at the ends, the bending stresses were probably higher than in the specimens of the other types. The conclusion seems justified that the maximum stress is practically independent of the length of the angle for all of these specimens.

VI. CONCLUSIONS

Shelf angles for transferring loads from floor beams to columns in steel-frame buildings were arc-welded to H-section columns. The types of weld connections used were: Single weld at heel of shelf angle; welds at heel and toe of angle; welds at ends of angle; welds at heel and ends of angle; welds at heel, toe, and ends of angles; and welds at ends of angles fitted between the flanges of the H-section. The specimens were tested to destruction with the following results:

1. Each specimen failed through the weld. Therefore, the properties of the material in the column and in the angles had little or no effect upon the values of the maximum load.
2. For the specimens having welds only at the ends of the angles the average maximum stress in the welds increased as the width of the vertical leg of the angle was increased.
3. There were no definite indications that the width of the vertical leg of the angles affected the strength of the specimens.
4. Neither the thickness nor the length of the angles appeared to have much effect on the strength of the specimens.
5. Shelf-angle connections similar to those tested in this investigation, with reinforced fillet welds, will have a factor of safety of about

4 if they are designed with the following assumptions: (*a*) The forces act in the face of the column; (*b*) vertical fillet welds are stressed in shear; (*c*) horizontal fillet welds at the heel of the angle are stressed in tension; (*d*) horizontal fillet welds at the toe of the angle are stressed in compression, and (*e*) the allowable stresses are those given in the Code for Fusion Welding and Gas Cutting in Building Construction prepared by the American Welding Society.

WASHINGTON, July 8, 1930.